C6.5 Abutments

See the Office of Bridges and Structures web site for archived Methods Memos listed under articles in this section.

The Methods Memos for which policies have been partially revised and/or for which document references have been updated are noted as partially revised. Any obsolete Methods Memos that apply to this section are listed at the end.

C6.5.1 General

C6.5.1.1 Policy overview

C6.5.1.1.1 Integral

Parameter study and discussion, 16 August 2007

In order to adjust integral abutment policy to LRFD and the latest ISU research (Abendroth and Greimann 2005) a parameter study was conducted to determine the effects of bridge length, end span length, skew, and prebore depth. For PPCB bridges the study specifically worked with the new A-D and BTB-BTD beams with an end span of maximum beam length. For CWPG members the basic condition was taken to be an end span of 150 feet or the maximum end span that would result in a pile structural resistance at Structural Resistance Level – 1, the LRFD equivalent to a 6 ksi axial stress under service load design.

In LRFD there is a single check for combined forces (axial load and bending) rather than the two (stability and yield) in service load design. The parameter study included the LRFD combined forces check and a ductility check (Abendroth and Greimann 2005). Generally the LRFD combined forces check gave results less conservative but similar to those from the stability and yield checks in service load design. A different, more conservative way of evaluating the effects of pile skew (similar to Abendroth and Greimann 2005), however, gave results essentially the same as those for past parameter studies.

With the latest ISU recommendations for ductility, the ductility check generally will not control the design, but use of the recommended seismic plate ratios requires that several H-pile shapes be avoided. Ratios for flange plates $b_f/2t_f$ above 11.0 do not work for Grade 50 steel, and the policy recommendation is to set an upper limit of 10.5, but either limit results in the same list of acceptable H-piles: HP 10x57, HP 12x74, HP 12x84, HP 14x102, and HP 14x117. The HP 14x102 shape should be avoided because it generally is not readily available.

Because of the less conservative biaxial bending and ductility checks, bridges with minimal skew may have greater lengths than present policy allows. Limits other than bridge length may be appropriate, however. Considering the type and performance of present pavement joints, the maximum bridge length for zero skew was set for approximately 1.55 inches maximum movement each way, assuming that the bridge is fixed at mid-length. At the maximum bridge length the pavement joints should be of the CF-3 type [OD SRP RH-52 and RK-20]. At shorter bridge lengths the CF-2 or CF-1 joints should be used within the guidelines on the standard road plan [OD SRP RK-20].

In general, the parameter study verified the previous study conducted for the service load design manual. The information for the Bridge Design Manual tables, however, was modified to better fit the LRFD format.

Reference

Abendroth, R.E. and Greimann, L.F. (2005) *Field Testing of Integral Abutments, Final Report HR-399.* Center for Transportation Research and Education (CTRE), Iowa State University, Ames, Iowa. Available online at <http://www.ctre.iastate.edu/reports/hr399.pdf>.

LRFD Integral Abutment Example

Given: Four-span PPCB bridge, 105-120-120-105-foot spans, 450-foot length, 20-degree skew
Five-BTC cross section, beam spacing 9'-3
Soil profile: 30 feet stiff silty clay, N = 6; sound bedrock, N = 210
Soils Design Section recommendation: end bearing on rock, maximum allowable in soils chart
Total abutment factored vertical load (includes IM) = $\Sigma\eta_i\gamma_iP_i$ = 1200 kips
Use HP 10x57 for integral abutment.

Nominal structural resistance for HP 10x57 at SRL-2, maximum in end bearing: P_n = 365 kips [BDM Table 6.2.6.1-1]. Note, however, that this maximum may not be permissible based on integral abutment limits, which may be less than SRL-2 [BDM Table 6.5.1.1.1-1].

Check maximum bridge length. Interpolate for 20-degree skew [BDM Table 6.5.1.1.1-1].

L_{max} = 525 + [(20-15)/(30-15)](475-525) = 508 feet; 508 feet > 450 feet, OK

Check integral abutment limit on nominal structural resistance.

Table 6.5.1.1.1-1 indicates that interpolation will not lead to 365-kip resistance, but shorter-than-maximum end span will permit some increase in extrapolated value.

Try 10-foot prebore with interpolation for skew; extrapolate for resistance with 120-foot end span.

P_n = 324 + [(20-15)/(30-15)](243-324) = 297 kips

Increase P_n for shorter-than-maximum end span.

P_n = (120/105)(297) = 339 kips, which is close to 365 kips. (Using a 15-foot prebore would permit the full 365 kips but, as the next step shows, the additional prebore would not reduce the number of piles.)

Determine number of piles

Number of piles, n = $\Sigma\eta_i\gamma_iP_i/\varphi_cP_n$ = 1200/(0.6*339) = 5.9, use 6

Check minimum: 5 beams require 5 piles, OK; maximum pile spacing is 8 feet, <u>use 6.</u>

Plan sheet bearing based on SRL-2 = 75*(339/365)(5.9/6) = 68.50, say <u>69 tons</u> [BDM Table 6.2.6.1-1]

By observation geotechnical resistance will be more than adequate. No drivability analysis is required during design because the piles have been limited to Structural Resistance Level - 2.

CADD Note E820 on plans: THE DESIGN BEARING FOR THE ABUTMENT PILES IS 69 TONS.

Methods Memo No. 79: Integral Abutment Piles
24 July 2003

<u>Memo 6.5.1.1.1 and 6.5.1.1.2-2011 ~ Abutment Backfilling at MSE Walls</u>
<u>During construction of the I-235 overpasses the Soils Design Section and Office of Construction decided not to place the abutment backfill sand with the flooding method given on standard sheets [OBS SS 1007D, 1007E] when the abutment was near an MSE wall. The primary reason was that the flooding water did not flow through the abutment subdrain. In 2011 the question of backfill flooding again was asked for the Wesley Parkway Bridge over I-29. The decision reached for the bridge and for standard practice was that when the abutment is within the MSE reinforced zone, flooding should not be used. The usual geotextile fabric, porous backfill, and abutment subdrain</u>

should be placed to divert deicer chemicals from the MSE wall straps. The abutment backfill should be the same material as placed for the MSE wall, and it should be placed in lifts and compacted in the same way as the MSE wall backfill material. Site constraints may dictate that the abutment subdrain be tied into the MSE wall subdrain. The designer will need to include a note on the plans that prohibits flooding of the backfill.

Methods Memo No. 86: New Policy for Bridge Approach Slabs
23 October 2003

Methods Memo No. 93: Approach Slab Responsibilities with Downdrag
31 March 2004

C6.5.1.1.2 Stub

Methods Memo No. 86: New Policy for Bridge Approach Slabs
23 October 2003

Methods Memo No. 93: Approach Slab Responsibilities with Downdrag
31 March 2004

Methods Memo No. 195: Stub Abutment Design Behind MSE Walls. Revision to Article 6.5.1.1.2 LRFD Bridge Design Manual
1 October 2008

Memo 6.5.1.1.1 and 6.5.1.1.2-2011 ~ Abutment Backfilling at MSE Walls
During construction of the I-235 overpasses the Soils Design Section and Office of Construction decided not to place the abutment backfill sand with the flooding method given on standard sheets [OBS SS 1007D, 1007E] when the abutment was near an MSE wall. The primary reason was that the flooding water did not flow through the abutment subdrain. In 2011 the question of backfill flooding again was asked for the Wesley Parkway Bridge over I-29. The decision reached for the bridge and for standard practice was that when the abutment is within the MSE reinforced zone, flooding should not be used. The usual geotextile fabric, porous backfill, and abutment subdrain should be placed to divert deicer chemicals from the MSE wall straps. The abutment backfill should be the same material as placed for the MSE wall, and it should be placed in lifts and compacted in the same way as the MSE wall backfill material. Site constraints may dictate that the abutment subdrain be tied into the MSE wall subdrain. The designer will need to include a note on the plans that prohibits flooding of the backfill.

C6.5.1.2 Design information

C6.5.1.3 Definitions

C6.5.1.4 Abbreviations and notation

C6.5.1.5 References

C6.5.2 Load application

C6.5.2.1 Dead

Methods Memo No. 57: Abutment Piling Design, PPCB Bridges
5 November 2001

C6.5.2.2 Live

Methods Memo No. 57: Abutment Piling Design, PPCB Bridges

5 November 2001

C6.5.2.3 Dynamic load allowance

C6.5.2.4 Centrifugal

C6.5.2.5 Braking force

C6.5.2.6 Earth pressure

C6.5.2.7 Live load surcharge

C6.5.2.8 Earthquake

C6.5.3 Load application

C6.5.3.1 Limit states

C6.5.3.2 Integral abutments

C6.5.3.3 Stub abutments

C6.5.4 Abutment analysis, design, and detailing

C6.5.4.1 Integral abutments

C6.5.4.1.1 Analysis and design

Partially revised: Methods Memo No. 14: Prebore Length for Integral and Stub Abutments
13 September 2001

Partially revised: Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009

C6.5.4.1.2 Detailing

Methods Memo No. 107: Integral Abutment and Pier Cap Detailing
6 June 2005

Methods Memo No. 52: Use of p3 Bars in Integral Abutments
18 October 2001

Methods Memo No. 105: Use of Epoxy-Coated Reinforcing Steel
28 March 2005

Methods Memo No. 86: New Policy for Bridge Approach Slabs (Modified by MM No. 93)
23 October 2003

Methods Memo No. 93: Approach Slab Responsibilities with Downdrag (Modification to MM No. 86)
31 March 2004

C6.5.4.2 Stub abutments

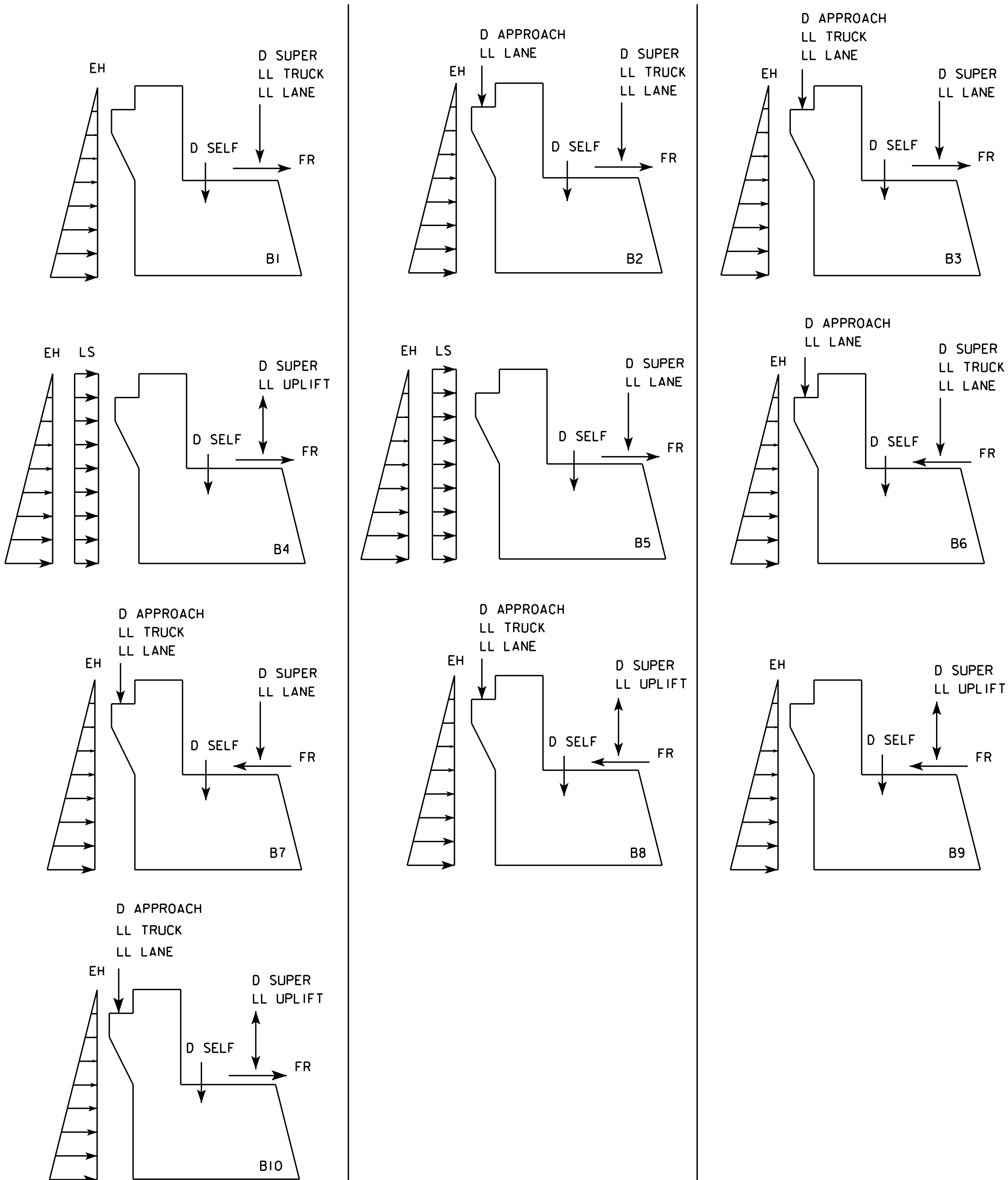
C6.5.4.2.1 Analysis and design

The following two figures for stub abutment load cases illustrate the typical cases that the designer should consider. The cases shown are not necessarily all of the cases to be considered for a specific bridge, and the designer should be on the alert for load cases to add or remove based on the bridge under design.

SERVICE I LOADINGS

CASES B1 TO B5: MAXIMIZE LOADS TO FRONT PILES, MINIMIZE BACK PILES.
CASES B6 TO B8: MAXIMIZE LOADS TO BACK PILES, MINIMIZE FRONT PILES.
CASE B9: MINIMIZE LOADS TO FRONT PILES.
CASE B10: MAXIMIZE SHEAR & MINIMIZE LOADS TO FRONT PILES. (ALSO B4)

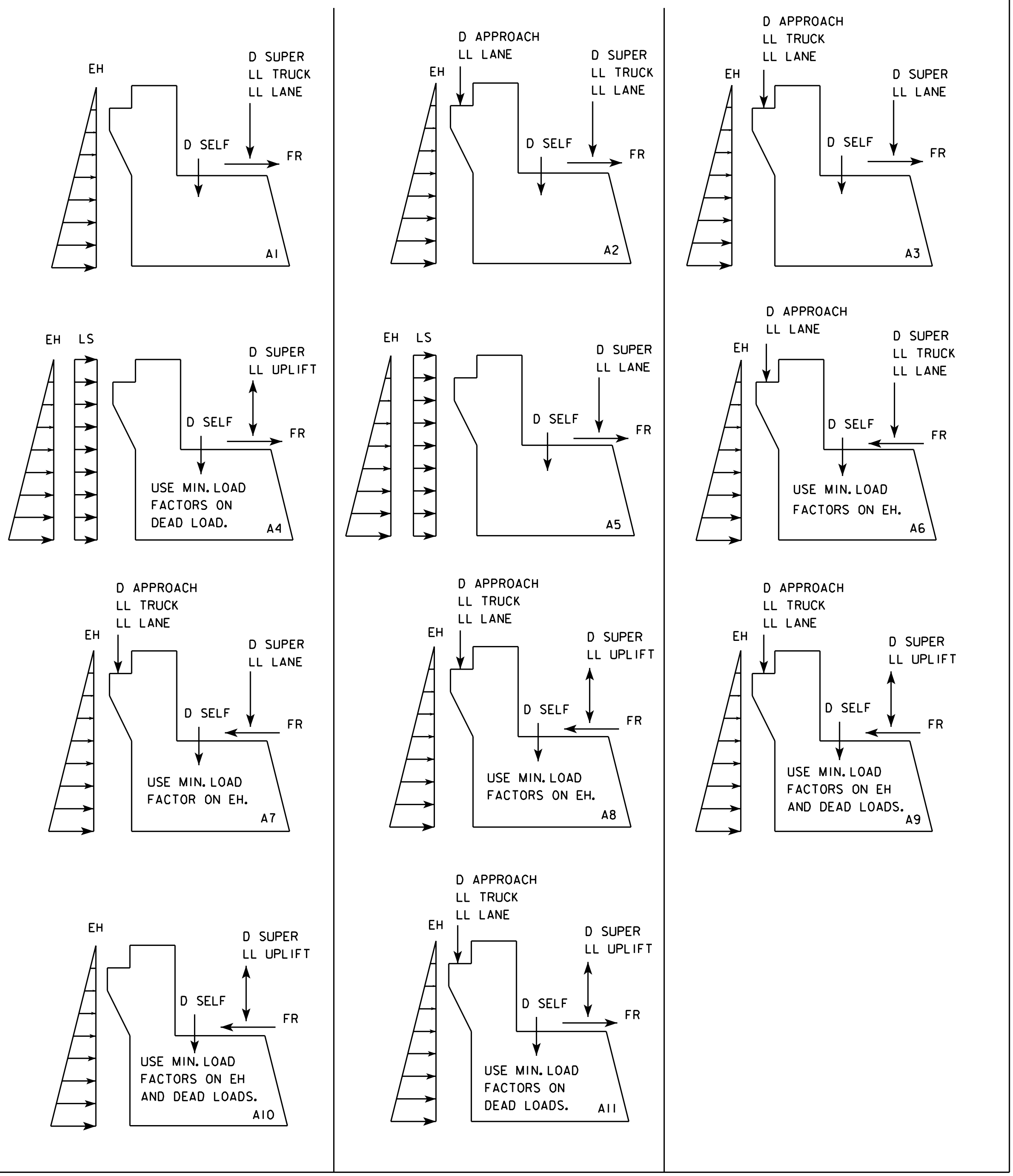
∴ LOAD FACTORS ARE 1.00



STRENGTH I LOADINGS

CASES A1 TO A5: MAXIMIZE LOAD TO FRONT PILES, MINIMIZE BACK PILES.
CASES A6 TO A8: MAXIMIZE LOAD TO BACK PILES, MINIMIZE FRONT PILES.
CASES A9 TO A10: MINIMIZE LOAD TO FRONT PILES
CASE A11: MAXIMIZE SHEAR & MINIMIZE LOAD TO FRONT PILES. (ALSO A4)

∴ MAX LOAD FACTORS USED UNLESS NOTED.



Partially revised: Methods Memo No. 211: Office Guidelines for Mass Concrete and Temperature and Shrinkage Reinforcing
1 September 2009

C6.5.4.2.2 Detailing

Methods Memo No. 105: Use of Epoxy-Coated Reinforcing Steel
28 March 2005

Methods Memo No. 86: New Policy for Bridge Approach Slabs
23 October 2003

Methods Memo No. 93: Approach Slab Responsibilities with Downdrag
31 March 2004

C6.5.4.3 Wing walls

C6.5.4.3.1 Analysis and design

Partially revised: Methods Memo No. 121: Use of Special Concrete Mixes on Bridges
8 July 2005

Methods Memo No. 33: Wing Extensions for C-Beams
11 July 2001

C6.5.4.3.2 Detailing

Methods Memo No. 105: Use of Epoxy-Coated Reinforcing Steel
28 March 2005

Obsolete: Methods Memo No. 23: Length Limits and Prebore Depths for Integral Abutment Bridges
30 October 2002 (Edited 29 January 2003)

Obsolete: Methods Memo No. 116: Correction to Figure 6.5.2.5 in 6.5 Abutments of the Bridge Design Manual
24 March 2005